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DESIGN OF THE KINGSTON BRIDGE OVER THE HUDSON RIVER

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Members, ASCE

STRUCTURAL DIVISION

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DESIGN OF THE KINGSTON BRIDGE OVER THE HUDSON RIVER

W. E. Joyce[1] and C. H. Gronquist,[2] Members, ASCE

INTRODUCTION

The Kingston Bridge will be the sixth highway structure below Albany (Fig. 1) over the bridge-studded river which got its name from Henry Hudson. New structures are planned as well for Newburgh and above Catskill near Selkirk, which will make east-west travel easy and convenient. When the program is completed it is expected that there will be a crossing every fifteen to twenty miles above the George Washington Bridge.

In 1947 the New York State Bridge Authority was authorized by the State Legislature to make a study as to the necessity for a Hudson River crossing near Kingston and, if such a crossing was found to be in the public interest, to determine its type and location.

D. B. Steinman, Consulting Engineer, New York was retained to make this study and in August 1948 presented a report to the Bridge Authority recommending a suspension bridge with a main span of 1700 feet located at Kingston Point.

Location

Upon presentation of the Report to the State Department of Public Works by the Bridge Authority, the necessity of the bridge was approved but the location was changed to the present site, (Fig. 2) which is approximately three and one-half miles north of Kingston. This location was chosen as best suited for a future east-west connecting route between the Taconic State Parkway and the new Thruway, the main north and south traffic arteries on the east and west sides of the Hudson River, respectively.

At the site the river is about 4400 feet in width, shore to shore, and has two navigable channels with a shoal area 1600 feet to 1700 feet wide near the middle of the river and separating the channels. Presently only the west channel is dredged.

The clearance requirements of the U. S. Army Engineers were for a clear width of 760 feet and a vertical clearance of 135 feet minimum for each of the two channels.

Type

Several types of bridges were studied for this location including through-cantilever trusses, tied-arch trusses, suspension spans in various arrangements, and continuous deck trusses. The most economical was the through-cantilever type, but for aesthetic reasons, for rigidity, for simplicity of detail and for its advantages to the motorist, the continuous deck type with arched

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bottom chord for the channel spans, was chosen as most preferable. The deck type of construction reduces driving hazards for the motorist and will allow him to enjoy an unobstructed view of this very beautiful section of the Hudson River Valley. The simple, graceful lines of the structure give it a pleasing and harmonious appearance, (Fig. 3).

Layout

The structure was originally designed having a four-lane roadway with a two-foot center mall and two emergency walks two feet wide. In 1953, for economic reasons, the engineers were directed to reduce the width of roadway and the final design has a three-lane, 36-foot roadway with a two-foot emergency walk on each side.

Grades on the east and west viaducts are 4% and these are connected by a 5200-foot parabolic vertical curve over the entire length of river spans.

The west viaduct, which is largely on a horizontal curve with a radius of 2656 feet, has simple plate girder spans for a total length of 1472 feet, while the east viaduct is on a tangent and has continuous girder spans for a total length of 1091 feet. The total length of the bridge is 7793 feet.

With spans over the channels fixed at 800 feet center to center of piers, (Fig. 4) the most economical arrangement of river spans was two continuous structures of 500'-800'-500' spans joined by a double 500' span continuous truss. At the east and west ends of the continuous spans, a 300' span simple truss is used.

Large concrete pylon piers, monumental in size, join the deep truss spans with the shallow viaduct spans and provide architectural punctuation at these points.

The roadway will be lighted by high-mounted mercury vapor lamps on aluminum standards set in the railings and alternately spaced 125 feet apart.

Substructure

Foundations

Subsurface soils investigations in the river section showed bedrock to vary in depth below mean low water from 75 feet to 149 feet. The material from the bed of the river to the rock is silt and clay, but on the land approaches there are outcroppings of rock alternating with deep beds of clay.

All foundations go to rock or rest on bearing piles driven to rock.

River Piers

The river piers (Fig. 5) are reinforced concrete bents supported on identical bases, except for the depth of tremie concrete to be placed in open sheet pile cofferdams. All piers have the same 1 in 48 batter for the outer faces of the columns and all piers have the same cross-section for the columns at Elevation plus 20, with the cross-section at the top of the columns varying with the height of the pier.

The tallest piers at the center of the crossing have a top portal strut and two intermediate cross struts, while the four lower piers for the 800' spans have a top portal strut and one intermediate cross strut. The piers vary in height from 129 feet to 194 feet above mean low water.

Each river pier has an open shaft 3 feet in diameter in the north column for a ladder from the top of the pier to the top of the pier base at Elevation

plus 20 to give access for maintenance of the navigation lights or fog bell.

The center pier is located on the shoal near the middle of the river, and hence the tremie seal is only 9 feet deep, with the bottom at Elevation minus 18. For Pier 20, which is the deepest and in 43 feet of water, the tremie seal is 20 feet deep with the bottom of the seal at Elevation minus 54. All concrete seals are placed on a 2-foot blanket of sand and gravel. Each pier base is faced with granite masonry from Elevation minus 4 to Elevation plus 20.

All river piers have 14" H 89# steel bearing piles with reinforced points. Allowable pile loads vary from 118 tons normal to a maximum of 177 tons during erection. The normal unit stress on the pile section is 9000 p.s.i., but this is decreased to 6000 p.s.i on the reinforced point.

Land Piers

The west abutment and five west viaduct piers rest on cast-in-place concrete bearing piles of 12" constant diameter. The remaining piers are founded on rock.

On the east viaduct two piers near the abutment rest on the cast-in-place concrete bearing piles and the remaining six piers and the abutment go to rock.

The viaduct piers (Fig. 6) are two-legged reinforced concrete bents with a top portal strut, except for the lowest piers where the top strut is omitted. The pier columns are 29 feet apart for the girder spans.

In order to eliminate falsework for support of the struts during concreting, a structural steel truss (Fig. 7) designed for this load is used as the permanent reinforcing for the struts.

Pier 13 on the west side supporting the outer end of the 300' truss span is a special pier with the columns 38 feet on centers and with a top portal strut and one intermediate strut similar to the river piers. This pier is founded on rock. The highest land pier is 128.5 feet above footing. Allowable loading for cast-in-place concrete piles is 40 tons, with a maximum of 50 tons for certain loading conditions.

Superstructure

Truss Spans - Cross-Section

The concrete floor slab will be 7-1/2" thick and both curbs and walkways will be of concrete. The trusses will be spaced 38 feet apart (Fig. 8) with stringers 9'3 "on centers and continuous over the floorbeams, which are spaced about 40 feet apart in the truss spans.

Design of the trusses is somewhat unusual in that the slab is supported directly on the top chord of the main trusses. Although slab construction joints are spaced at from 75 feet to 125 feet, the sliding stringer joints used to reduce participation stresses are widely spaced at from 160 feet to 330 feet in order to minimize the number of joints in the roadway. This construction tends to make composite action between the slab and the chords, as well as the stringers, more complete, although no advantage is taken of this to reduce the steel sections. Slab anchors are attached to the chords and stringers to force the stringers to deform with the chords to a great extent, in order to reduce floorbeam longitudinal bending under live load. For this purpose the stringers are specified to be shop detailed to chord length under full dead load. The order of concreting of the deck is arranged to minimize any tendency of the slab to shrink or to crack from composite action

during concreting, by the requirement that the concrete over the tension top chords shall be the last to be placed.

Continuous Truss Layout

The 5200-foot length of truss spans is made up of the 300' simple span at each shore end and adjacent to them the 500'-800'-500' continuous truss unit over each of the two river channels. These units are connected by the 500'-500' span continuous truss. The depth of the trusses is 50 feet in the 300' simple span and in the double 500' continuous unit. For each 500'-800'-500' unit the depth of truss varies from 50 feet at the outer ends of the 500' span to 90 feet at the ends of the 800' span and 55 feet at the center of that span, the bottom chord lying in a parabolic arch in the 800' span and in half arches in the 500' spans.

The trusses will be set on fixed bearings at both ends of the 800' spans and at all three piers of the double 500' spans for simplicity of detail and to distribute the longitudinal forces to all piers. The trusses and the piers are designed considering the temperature, live load and wind forces which result from this use of fixed bearings.

Design Loads

The floor system is designed for H20-S16 live loading, in accordance with the 1953 specifications of the American Association of State Highway Officials. The trusses are designed for H20-S16 live load, in accordance with the recommendations of the Joint Committee on Live Loading for Long Span Highway Bridges for reduction in intensity of loading for length of loading exceeding 600 feet. For this bridge the effect of this loading is to reduce the total design stresses by an average of 3% below those which would be obtained from AASHO loading. The stresses due to uniform temperature change and allowance for adjustment of reactions equivalent to one inch settlement of any pier are combined with the dead load stresses. Wind stresses are computed for 50-pound wind in combination with dead load, or for 15-pound wind in combination with dead load and live load, in accordance with the AASHO specifications. However, the wind does not govern the design of the truss members when 25% overstress is permitted for these stress combinations.

Proportions of Truss Members

All truss members are designed with the four angles turned in to obtain the good appearance of flush surfaces and for simplicity of detail. Perforated cover plates top and bottom with 9-inch minimum width of perforation are used throughout, except in the top chord where a full top cover plate helps in taking the bending moment resulting from slab dead load and distributed live load. The chords are 30" deep and 24" wide, with the heaviest section, which occurs in the top chord over the piers at the ends of the 800' span, having a gross area of 220 square inches of silicon steel. The diagonals vary in width from 18" to 30" and the verticals are, in general, 15" wide, except over the intermediate piers (Fig. 9) where their width is 30". Lug angles are used to transfer the stress from the perforated cover plates to the gusset plates where the perforated cover plates are thicker than 1/2". For lighter perforated plates their stress is transferred to the gusset plates through the rivets in the main angle.

Truss Splices

The chord splices are located on the gusset plates for economy, for stiffness, and for ease of erection. When the thickness of splice material could be determined by stress, the lever-arm method was used for computation of the splice, but when the thickness of splice material was determined by the necessity for matching gusset plate or web plate thickness, the uniform stress method was employed. All truss members are specified to be milled at both ends to secure accuracy of layout and good fit. However, all compression chord splices are designed for 100% of capacity of member since the accuracy of milling is sometimes questionable. The chord splices are to be reamed assembled after check for length and offset in total length of layout of at least 300 feet. In order to reduce secondary stresses as far as practicable, all truss joints will be detailed to make the axes of the members conform to the dead load layout of the truss. The reaming of connections for the web members will be done to metal templets.

Indeterminate Stress Analysis

For the analysis of the continuous trusses, the redundants were taken as the stress in the tension chords over the intermediate piers, corresponding to a redundant moment at the intermediate supports. By the method of elastic weights, the vertical deflection was obtained at each panel point of each span, treated as a simple span, for a unit moment at the intermediate end of span. The deflection at each panel point for a unit moment at end of simple span will equal the rotation at the point of application of the unit moment due to a unit load at the panel point.

For the three-span continuous truss, an equation was set up for each of the two intermediate supports expressing the relation of the rotations at each support. By solving these two equations, the value of the moment at the intermediate support for a unit load at each panel point was obtained. For the two-span continuous truss, only one redundant was involved and the application of the same method was extremely simple. The influence line ordinates were obtained directly by computing the vertical deflections for a unit moment at the intermediate support.

Other methods of indeterminate analysis, such as moment distribution, are also advantageous in the solution of this problem, but a check by that method indicated no great saving in time could be realized.

Erection of Truss Spans

It is expected that erection of the truss spans will commence at the shore end of the 300' span and that it will proceed, with the use of a deck traveller, by cantilevering over several falsework bents to the next pier.

Erection will continue with the use of three bents in the same manner in the first 500' anchor span of the 500'-800'-500' unit and balanced erection will proceed in both directions from the second intermediate pier of this unit with the use of two falsework bents in the 500' span. Erection of the 800' span will be done by cantilevering to the center of span without the use of falsework bents.

After the second half of the 800' span has been cantilevered from the second intermediate pier to the center, it will be necessary to effect closure at that point. This will require jacking at the top chord and pulling at the bottom chord, both of which can be greatly minimized by erecting the trusses with the outer or free ends set low. The two ends of the top chord at the

center of span will be jacked apart a sufficient distance to enter an insert section. Following closure at the center, the ends of the unit would be raised by jacking to normal position and to introduce the design dead load stresses into the continuous truss.

Erection of the double 500' span continuous unit will also be done by cantilevering over several falsework bents to the piers. Moderate jacking will probably be required to connect the truss to the fixed bearings.

Bearings

The fixed bearings for the truss spans (Fig. 10) are designed as weldments with moderate use of ribs to economize on weight without requiring excessive size or undue amount of weld. The total truss reaction for the 800' span is 4535 kips, and the weight of this bearing is 24000 pounds.

The lighter viaduct bearings are weldments of simple design which eliminates distributing ribs without requiring excessively heavy vertical or horizontal slabs.

Grillages are used extensively for the truss bearings even when not required for distribution of load, but are employed to obtain accurate adjustment in setting of bearings.

Four-inch tie-down anchor bolts (Fig. 11) are provided at the expansion ends of the 500'-800'-500' continuous truss spans. These bolts, which occur in pairs, engage a yoke set over but not connected to the transverse strut. This arrangement will resist any tendency to uplift at the end of truss, although no computed uplift forces are obtained. The tie-down bolts may also be useful to resist possible uplift or for jacking during erection.

Lateral Systems for Trusses

Both top and bottom lateral systems are X-bracing and are designed as tension-compression systems as for K-bracing. Structural T-sections are used for the top laterals. For the bottom laterals, wide flange sections are used opposite compression chords and a built-up section consisting of two angles with shaped batten plates is used opposite tension chords. All laterals are designed for full dead load and live load participation stresses, in addition to wind stresses. The main sway frames at the piers are designed to transmit the wind carried by the top chord increased by one-half the wind from the bottom chord, since analysis indicated that half of the bottom chord wind load would be transferred by the intermediate sway frames to the concrete deck.

Truss Weights

The weights of carbon and silicon steel in the trusses, floor system and bracing per foot of bridge are as follows:

	500'-800'-500'	500'-500'
Trusses	3375	2700
Floor System	980	1000
Bracing	520	430
Total	4875	4130

Viaduct Cross-Section

The viaduct girders are spaced 29 feet apart (Fig. 12). The floorbeam brackets are 5'6" in length and have a tension plate which runs through a slot

in the main girder web at about one-third the bracket depth below top of girder. A nominal tieplate also connects the top of bracket to the girder.

The stringers are continuous over the floorbeams in 25' panel lengths and are spaced 9'3 "on centers. Slab construction joints and stringer joints occur over the piers, and construction joints are further used at the floor-beams on either side of intermediate piers so that the concrete over the portion of the top flange that is in tension may be the last concrete to be placed.

Viaduct Layout

Since the east viaduct is on tangent and the foundation conditions are suitable, the girders are continuous in three 3-span units. The lengths of span are three at 150 feet, having a girder depth of 10 feet; three at 125 feet, having a girder depth of 8 feet 6 inches; and three at 87 feet, having a girder depth of 7 feet.

The simple spans in the west viaduct on curve are ten at 125 feet and two at 100 feet, all having a girder depth of 9 feet 6 inches.

Carbon steel is used for all viaduct girders. The flanges for the 150' and 125' girders have two 8" x 8" angles with 22" cover plates. Transition in depth of girders in the east viaduct is obtained by the use of brackets at bottom of shallow girder to receive the bearing, in order to avoid stepped piers.

Concrete Deck

The 7-1/2" concrete slab provides for a 1/2" thickness of integral wearing surface. Reinforcing steel is straight throughout without bends, in accordance with recent practice for ASTM A305 special deformed bars. The reinforcement is intermediate grade steel and was designed in accordance with 1949 AASHO specifications. Additional longitudinal reinforcement is provided over tension chords and flanges, and also over all floorbeams to resist any tendency to slab cracking due to composite action or continuous stringer deflection. Screeding is specified to be done with the use of a vibratory power screed of the portable type which is hand-pulled in a longitudinal direction.

Fascia and Railing

On the viaduct the fascia girder is load-carrying and is continuous. It has the same 30" depth and channel form as the exterior of the truss top chord. The fascia proper, used to face the concrete for the depth of walkway, is flush and is offset six inches to enclose the steel railing posts which are spliced at top of walk. The railing is of horizontal type of construction with three steel channel rails, and the 6" H-railing posts are spaced between six and seven feet on centers.

Personnel

The bridge is being built by the New York State Bridge Authority. The members of the Bridge Authority are Robert Hoe, Chairman; John L. Edwards, M.D., Vice Chairman; Neal Brandow, Harry Cohen and Jay LeFevre. Mr. William K. Haggenbothom is Executive Officer.

The late Mr. James F. Loughran, C. E., Member A.S.C.E., of Kingston, New York, was Chairman of the Bridge Authority from 1947 until his death in May 1954.

D. B. Steinman is Consulting Engineer for design and supervision of construction of the bridge. W. E. Joyce is Project Engineer, Ray M. Boynton is

in charge of design of the substructure, Carl H. Gronquist is in charge of the design of the superstructure, and Milton O. Elkow is Resident Engineer for construction. Members of the staff in a supervisory capacity are A. Werth, A. Zuckerman, and I. S. Hansen.

Contract for the substructure has been awarded to the Merritt-Chapman & Scott Corporation for the sum of $4,495,477.00, and contract for the superstructure has been awarded to the Harris Structural Steel Co., Inc. for the sum of $9,496,431.80. The total of these two bids is 5% lower than the engineer's estimate for that work. Highway connections at both ends of the bridge designed by D. B. Steinman are now under construction by the New York State Department of Public Works at a total estimated cost of $750,000.00. The total estimated cost of construction, including toll facilities, is $16,000,000.00. Bridge construction was commenced during the summer of 1954 and it is expected that the bridge will be opened to traffic in the fall of 1956.

The bridge is a toll project and is being financed with funds on hand and from the proceeds of sale of bonds by the New York State Bridge Authority.

Fig. 1. Map of Hudson River Valley.

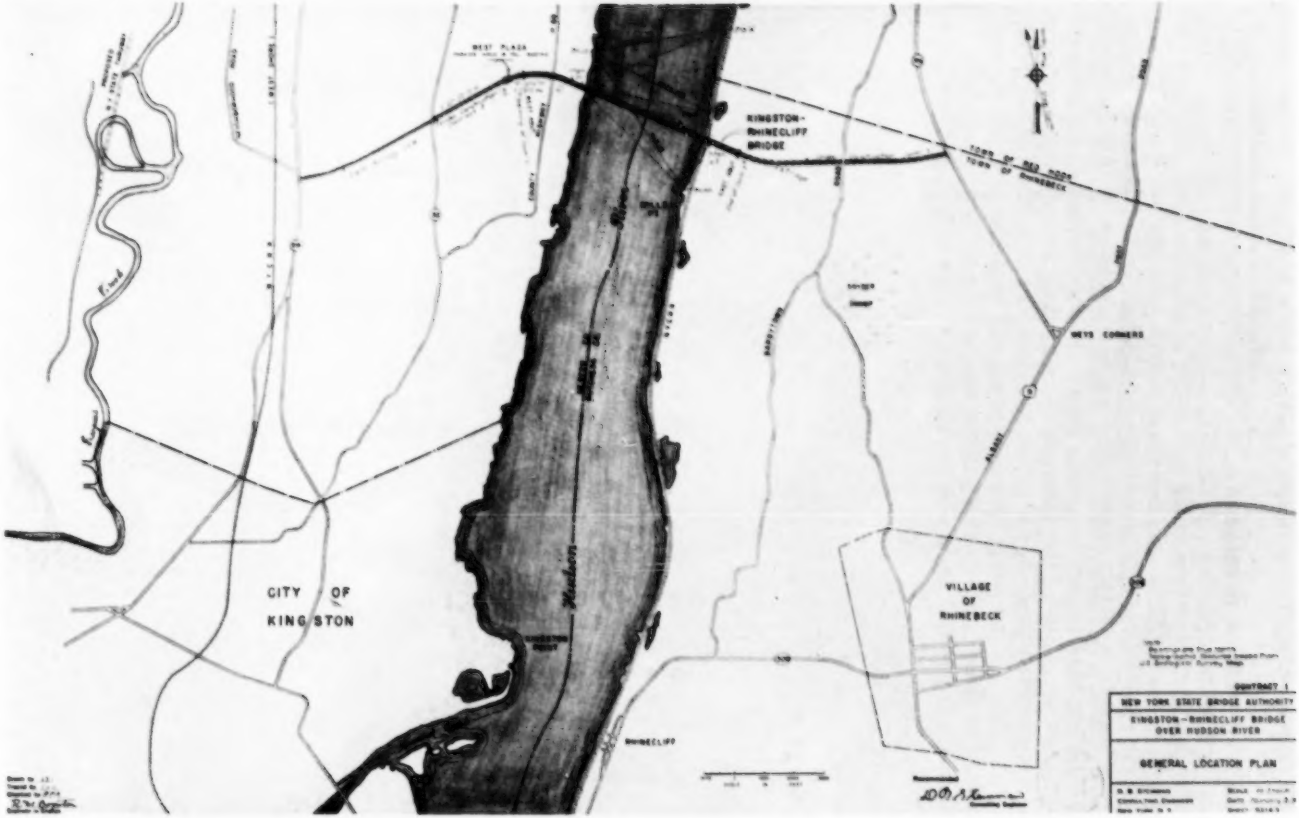


Fig. 2. Location Map.

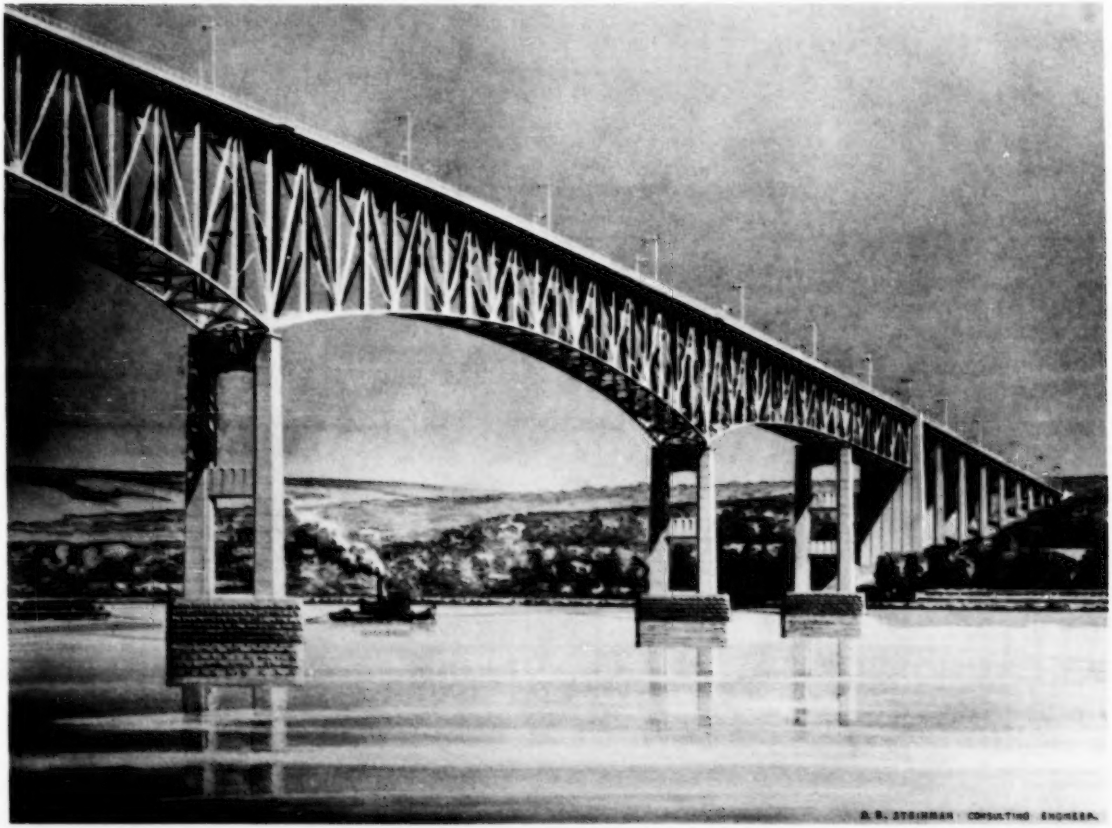
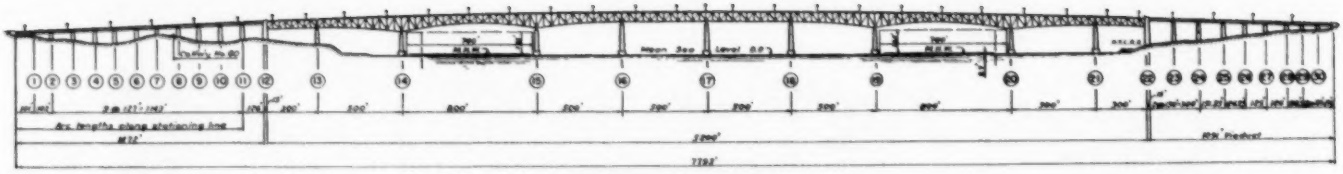


Fig. 3. Rendering of Bridge.



ELEVATION OF BRIDGE

PLAN OF BRIDGE

Fig. 4. General Plan and Elevation.

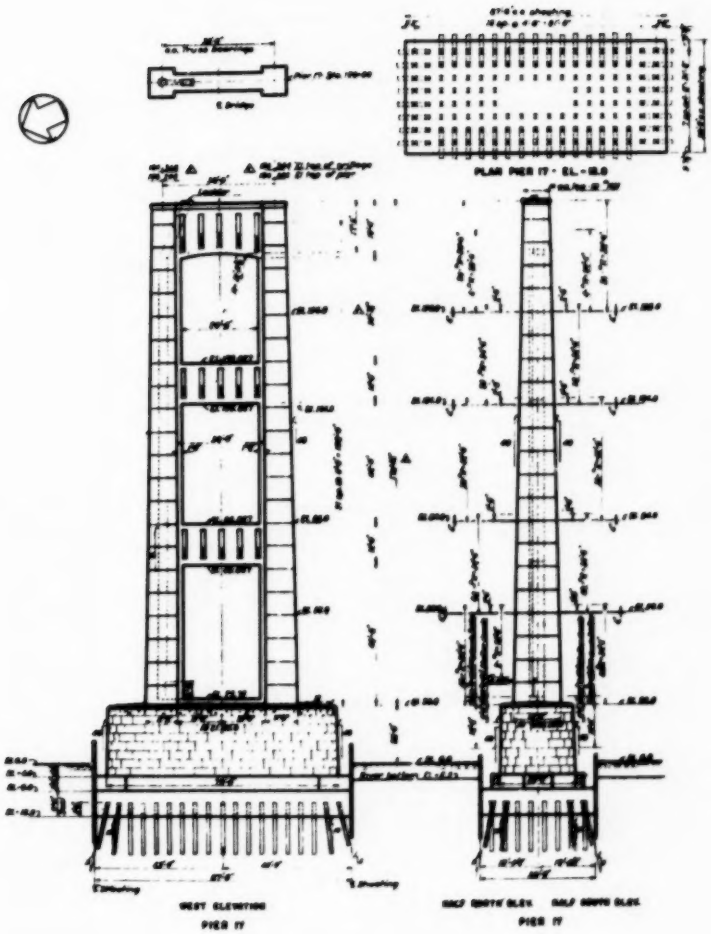


Fig. 5. River Pier 17.

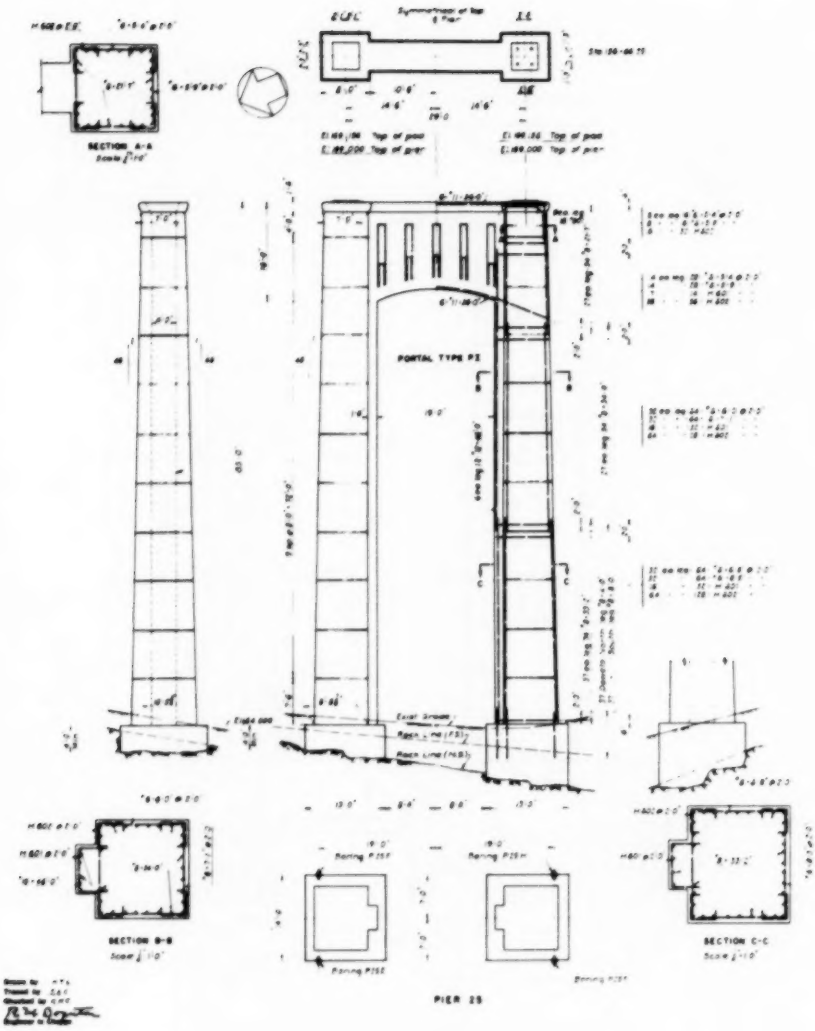


Fig. 6. Land Piers.

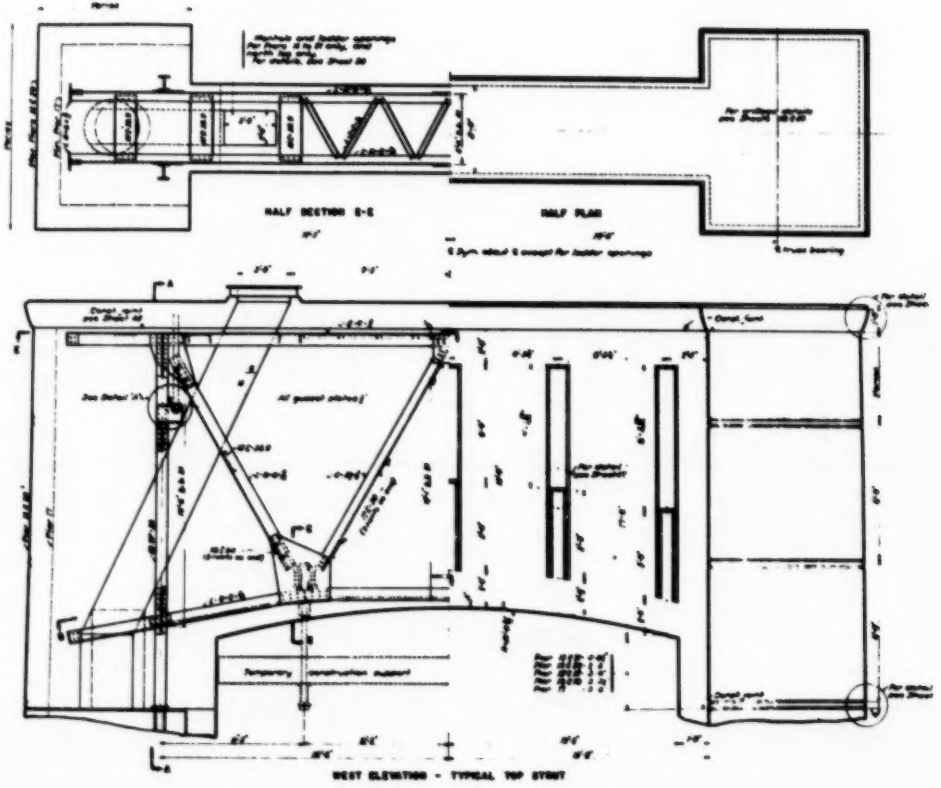
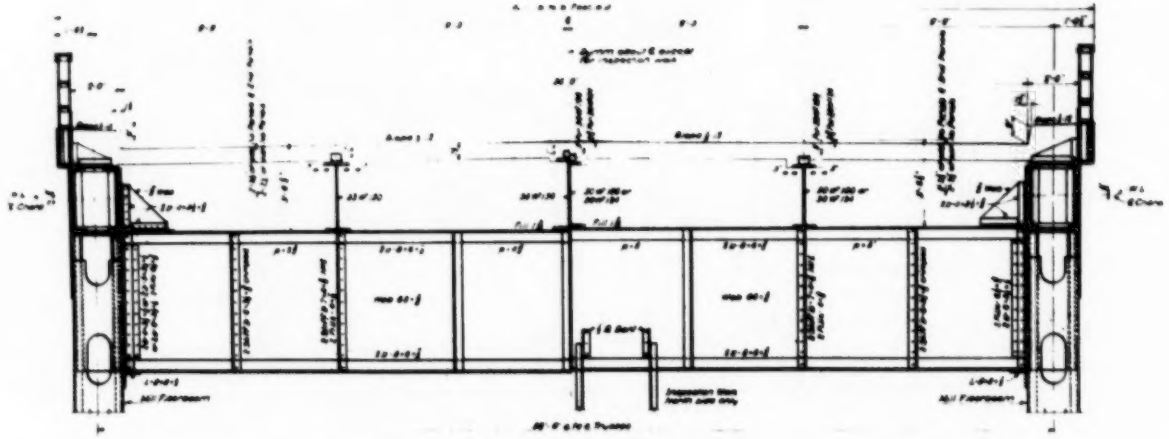


Fig. 7. Top Strut of Pier.



HALF SECTION FOR 498FT. & 800FT. SPANS

HALF SECTION FOR 298.5FT. SPAN

Fig. 8. Cross-Section of Truss Spans.

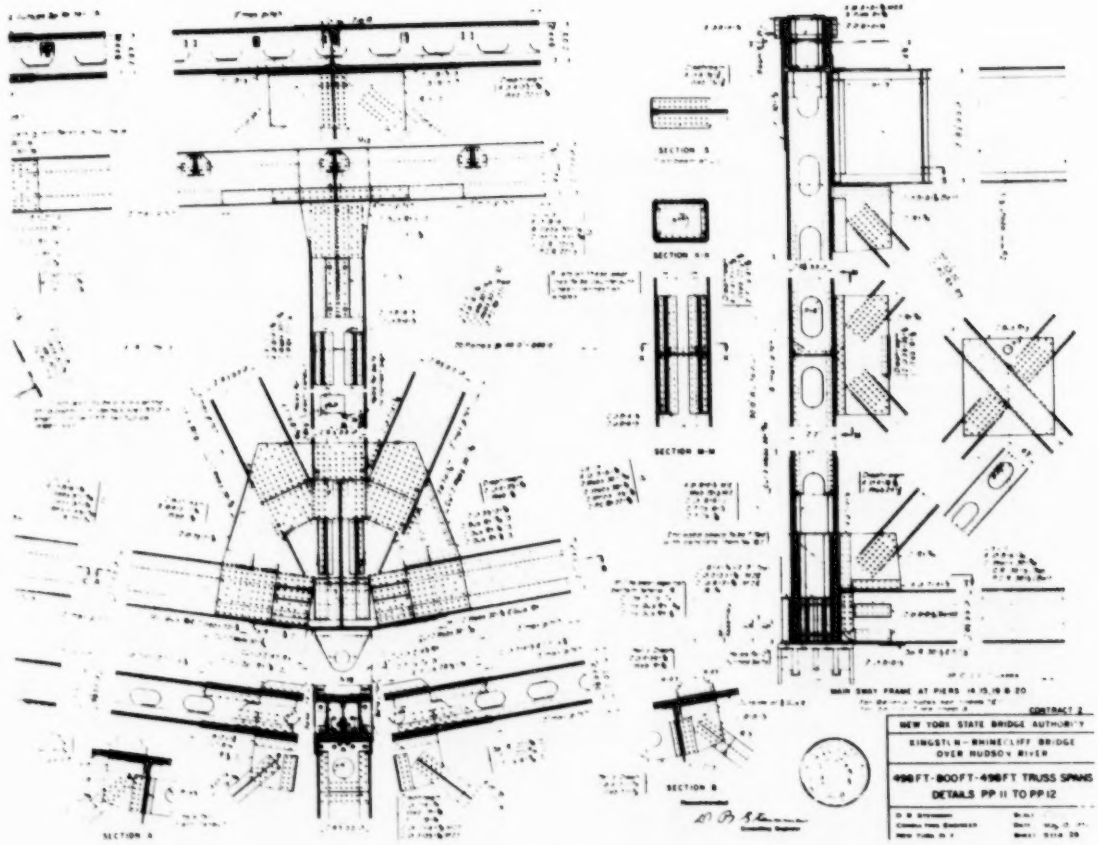

NEW YORK STATE BRIDGE AUTHORITY
KINGSTON-RHINECLIFF BRIDGE
OVER HUDSON RIVER
498 FT-800 FT-498 FT TRUSS SPANS
DETAILS PP 11 TO PP 12

Fig. 9. Truss Details at Intermediate Pier of 800' Span.

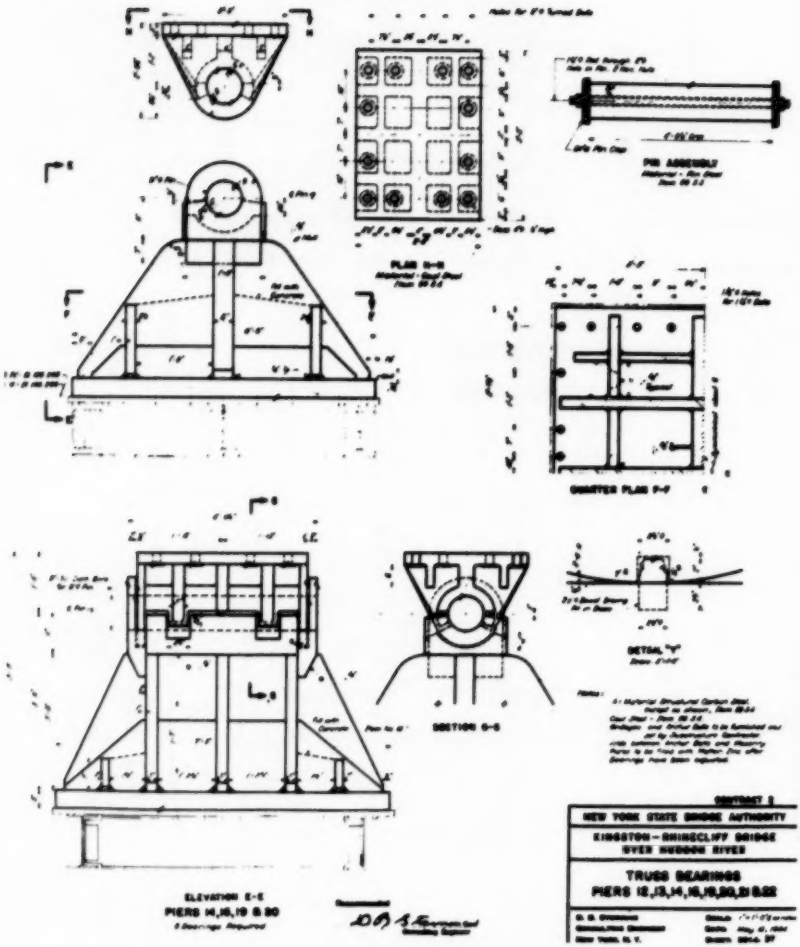


Fig. 10. Truss Bearings.

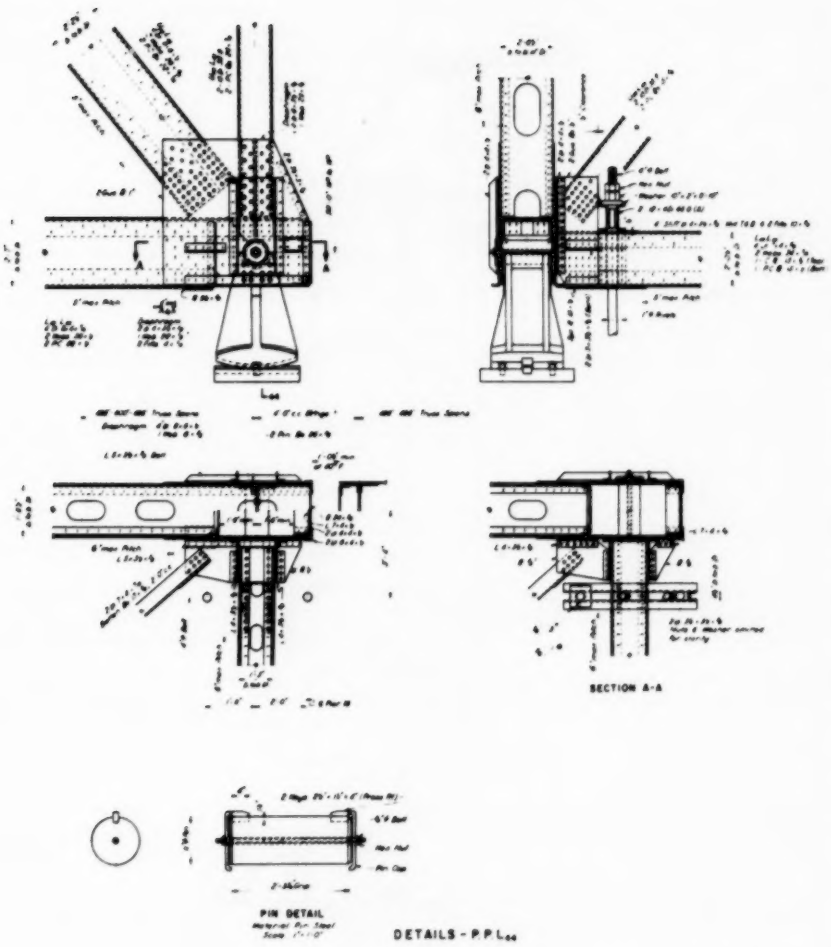


Fig. 11. End Tie-Down Details for 500' -800' -500' Unit.

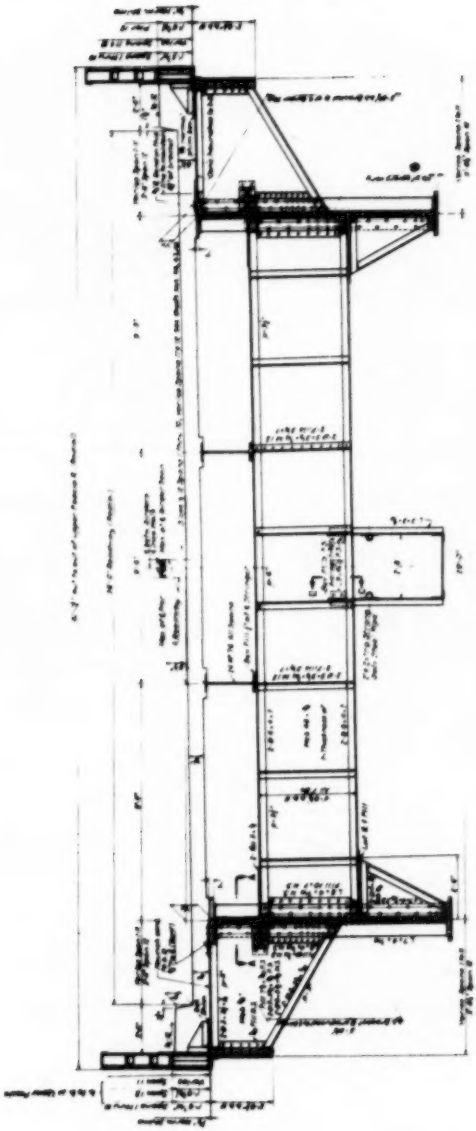


Fig. 12. Viaduct Cross-Section.

PROCEEDINGS-SEPARATES

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MAY: 435(SM), 436(CP)[c], 437(HY)[c], 438(HY), 439(HY), 440(ST), 441(ST), 442(SA), 443(SA).

JUNE: 444(SM)[e], 445(SM)[e], 446(ST)[e], 447(ST)[e], 448(ST)[e], 449(ST)[e], 450(ST)[e], 451(ST)[e], 452(SA)[e], 453(SA)[e], 454(SA)[e], 455(SA)[e], 456(SM)[e].

JULY: 457(AT), 458(AT), 459(AT)[c], 460(IR), 461(IR), 462(IR), 463(IR)[c], 464(PO), 465(PO)[c].

AUGUST: 466(HY), 467(HY), 468(ST), 469(ST), 470(ST), 471(SA), 472(SA), 473(SA), 474(SA), 475(SM), 476(SM), 477(SM), 478(SM)[c], 479(HY)[c], 480(ST)[c], 481(SA)[c], 482(HY), 483(HY).

SEPTEMBER: 484(ST), 435(ST), 486(ST), 487(CP)[c], 488(ST)[c], 489(HY), 490(HY), 491(HY)[c], 492(SA), 493(SA), 494(SA), 495(SA), 496(SA), 497(SA), 498(SA), 499(HW), 500(HW), 501(HW)[c], 502(WW), 503(WW), 504(WW)[c], 505(CO), 506(CO)[c], 507(CP), 508(CP), 509(CP), 510(CP), 511(CP).

OCTOBER: 512(SM), 513(SM), 514(SM), 515(SM), 516(SM), 517(PO), 518(SM)[c], 519(IR), 520(IR), 521(IR), 522(IR)[c], 523(AT)[c], 524(SU), 525(SU)[c], 526(EM), 527(EM), 528(EM), 529(EM), 530(EM)[c], 531(EM), 532(EM)[c], 533(PO).

NOVEMBER: 534(HY), 535(HY), 536(HY), 537(HY), 538(HY)[c], 539(ST), 540(ST), 541(ST), 542(ST), 543(ST), 544(ST), 545(SA), 546(SA), 547(SA), 548(SM), 549(SM), 550(SM), 551(SM), 552(SA), 553(SM)[c], 554(SA), 555(SA), 556(SA), 557(SA).

DECEMBER: 558(ST), 559(ST), 560(ST), 561(ST), 562(ST), 563(ST)[c], 564(HY), 565(HY), 566(HY), 567(HY), 568(HY)[c], 569(SM), 570(SM), 571(SM), 572(SM)[c], 573(SM)[c], 574(SU), 575(SU), 576(SU), 577(SU), 578(HY), 579(ST), 580(SU), 581(SU), 582(Index).

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FEBRUARY: 608(WW), 609(WW), 610(WW), 611(WW), 612(WW), 613(WW), 614(WW), 615(WW), 616(WW), 617(IR), 618(IR), 619(IR), 620(IR), 621(IR)[c], 622(IR), 623(IR), 624(HY)[c], 625(HY), 626(HY), 627(HY), 628(HY), 629(HY), 630(HY), 631(HY), 632(CO), 633(CO).

MARCH: 634(PO), 635(PO), 636(PO), 637(PO), 638(PO), 639(PO), 640(PO), 641(PO)[c], 642(SA), 643(SA), 644(SA), 645(SA), 646(SA), 647(SA)[c], 648(ST), 649(ST), 650(ST), 651(ST), 652(ST), 653(ST), 654(ST)[c], 655(SA), 656(SM)[c], 657(SM)[c], 658(SM)[c].

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MAY: 679(ST), 680(ST), 681(ST), 682(ST)[c], 683(ST), 684(ST), 685(SA), 686(SA), 687(SA), 688(SA), 689(SA)[c], 690(EM), 691(EM), 692(EM), 693(EM), 694(EM), 695(EM), 696(PO), 697(PO), 698(SA), 699(PO)[c], 700(PO), 701(ST)[c].

c. Discussion of several papers, grouped by Divisions.

e. Presented at the Atlantic City (N.J.) Convention in June, 1954.

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